REAL-TIME HYDRODYNAMIC MODELING OF COASTAL RIVERS

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INTRODUCTION

The flow in coastal rivers is complicated by the effects of the ocean, i.e., its dissipation of river flood waves, its tidal action, and wind generated storm surges. In streamflow forecasting, transient stages and discharges are computed for various forecast points along a river from a given (predicted or observed) stage or discharge hydrograph at either the upstream extremity of the river reach as in the case of a flood wave propagating in the downstream direction, or at the downstream extremity as in the case of a tidal or hurricane surge propagating in the upstream direction. The National Weather Service (NWS) provides real-time forecasts of the unsteady flows and water surface elevations in many rivers throughout the Nation including such coastal rivers as the Mississippi, Columbia, Sabine, Neches, and Trinity rivers. To compute these stages and discharges, a one-dimensional, implicit hydrodynamic model (DWOPER) is used. This paper presents a brief description of the DWOPER model and how it utilizes a river-ocean interfacial boundary for each of the above oceans effects. Model applications, efficiency, and calibration are also discussed.

MODEL DESCRIPTION

Mathematical Basis. The DWOPER model is a generalized one-dimensional hydrodynamic model developed by Fread (1973, 1978, 1981) for use in river systems where simple storage routing methods are inadequate due to the effects of backwater, tides, and mild channel bottom slopes. The basis for the model is a finite difference solution of the conservation form of the one-dimensional equations of unsteady flow consisting of the conservation of mass and momentum equations, i.e.,

$$\frac{\partial Q}{\partial x} + \frac{\partial (A+A_o)}{\partial t} - q = 0 \tag{1}$$

$$\frac{\partial Q}{\partial t} + \frac{\partial (Q^2/A)}{\partial x} + gA\left(\frac{\partial h}{\partial x} + S_f + S_e\right) - qv_x + W_f B = 0 \tag{2}$$

where: $$S_f = \frac{n^2|Q|Q}{2.21\ A^2 R^{4/3}} \tag{3}$$

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$$S_e = \frac{K_e}{2g} \frac{\partial (Q/A)^2}{\partial x} \tag{4}$$

$$W_f = C_w (V_r \cos \omega)^2 \tag{5}$$

in which x is distance along the longitudinal axis of the waterway, t is time, Q is discharge, A is active cross-sectional area, A_o is inactive (off-channel storage) cross-sectional area, q is lateral inflow (+) or outflow (-), g is the gravity acceleration constant, h is water surface elevation, B is wetted topwidth of the cross section, v_x is the velocity of the lateral inflow in the x-direction of the main channel flow, S_f is friction slope computed from Manning's equation, n is the Manning roughness coefficient, R is the hydraulic radius approximated by (A/B), S_e is the local loss slope, K_e is an expansion (-) or contraction (+) coefficient, W_f is the wind term, C_w is non-dimensional wind coefficient, V_r is the velocity of the wind (V_w) relative to the velocity of the channel flow, and ω is the angle between the wind direction and channel flow direction.

In an implicit finite difference solution of Eqs. (1) and (2), the continuous x-t solution domain in which solutions of h and Q are sought is represented by a rectangular net of discrete points as shown in Fig. 1. The net points (nodes) may be at equal or unequal intervals of Δt and Δx along the t and x axes, respectively. Each node is identified by a subscript (i) which designates the x position and a superscript (j) for the time line. A four-point weighted, implicit difference approximation is used to transform the nonlinear partial differential equations of unsteady flow into nonlinear algebraic equations. The four-point weighted difference approximations are:

$$\frac{\partial K}{\partial t} = \left(K_i^{j+1} + K_{i+1}^{j+1} - K_i^j - K_{i+1}^j\right)/(2\Delta t^j) \tag{6}$$

$$\frac{\partial K}{\partial x} = \Theta/\Delta x_i \left(K_{i+1}^{j+1} - K_i^{j+1}\right) + (1-\Theta)/\Delta x_i \left(K_{i+1}^j - K_i^j\right) \tag{7}$$

$$K = 0.5\ \Theta \left(K_i^{j+1} + K_{i+1}^{j+1}\right) + 0.5(1-\Theta) \left(K_i^j + K_{i+1}^j\right) \tag{8}$$

where K is a dummy parameter representing any variable in the above differential equations, Θ is a weighting factor varying from 0.5 to 1, i is a subscript denoting the sequence number of the cross section or Δx reach, and j is a superscript denoting the sequence number of the time line in the x-t solution domain. A Θ value of 0.5 is known as the "box" scheme while $\Theta = 1$ is the "fully implicit" scheme. To insure unconditional linear numerical stability and provide good accuracy, Θ values nearer to 0.5 are recommended (Fread, 1974). Accuracy decreases as Θ departs from 0.5 and approaches 1.0. This effect becomes more pronounced as the time step size increases. DWOPER allows Θ to be an input parameter. A value of 0.55 to 0.60 is often used to minimize loss of accuracy while avoiding weak or pseudo instability when Θ of 0.5 is used. Accuracy of the weighted four-point scheme depends on the selection of Δt, i.e.,

$$\Delta t \leq 0.11\ c\ Z\ T_p/\sqrt{D_o} \tag{9}$$

where: $$Z = \left[\frac{1 - \varepsilon^2}{4\Theta^2 \varepsilon^2 - (2\Theta - 2)^2} \right]^{1/2} \tag{10}$$

in which Δt is the time step in hours, T_p is the time of rise of the flood wave in hours, c is the wave celerity in ft/sec, ε is one minus the permissible error(%) where $0.9 < \varepsilon < 0.99$, and D_o is the initial hydraulic depth (A/B) in ft. Thus, time steps are chosen solely on the basis of desired accuracy since the implicit finite difference technique is not restricted to the very small time steps of explicit techniques due to numerical stability considerations. This enables DWOPER to be very efficient as to computational time for simulating unsteady flows, particularly those with a very long time of rise.

Substitution of the finite difference quotients defined by Eqs. (6-8) into Eqs. (1) and (2) for the derivatives and non-derivative terms produces two algebraic equations which are nonlinear with respect to the unknowns h and Q at the net points on the j^{th} time line. All terms associated with the j+1 time line are known from either the initial conditions or previous computations. The initial conditions are values of h and Q at each computational point (node) along the x-axis for the first time line (j=1). They are obtained from a previous unsteady flow solution, or they can be estimated since small errors in the initial conditions dampen out within several time steps. Estimations are obtained from conditions at the beginning of the solution, e.g., interpolated values between observations obtained at gaging stations along the river, or assuming steady flow and computing the values of h via a backwater algorithm.

The two nonlinear algebraic equations cannot be solved in a direct (explicit) manner since there are four unknowns, h and Q, at points i and i+1 on the j+1 time line and only two equations. However, if similar equations are formed for each of the N-1 Δx reaches between the upstream and downstream boundaries, a total of 2N-2 equations with 2N unknowns results. (N denotes the total number of computational points or cross sections.) Then prescribed boundary conditions, one at the upstream extremity of the river and one at the downstream extremity, provide the necessary two additional equations required for the system to be determinate. The resulting system of 2N nonlinear equations with 2N unknowns is solved by a functional iterative procedure, the Newton-Raphson method (Amein and Fang. 1970). In the iterative procedure, trial values obtained via linear or parabolic extrapolation from solutions of h and Q at previous time steps are assigned to the 2N unknowns. Substitution of these into the system of 2N nonlinear equations yields a set of 2N residuals. The Newton-Raphson method seeks to reduce the residuals to an acceptable tolerance level which is usually achieved within one or two iterations.

The Newton-Raphson method generates a system of 2N x 2N linear equations. The coefficient matrix of the system is composed of partial derivatives which are functions of the unknowns; however, the elements in the coefficient matrix can be assigned numerical values by substituting in the trial values for the unknowns. The coefficient matrix is related to the set of 2N residuals by a set of 2N corrections to the original trial values of the unknowns. It is the 2N corrections that are sought in the solution of the 2N x 2N linear system. The coefficient matrix has a banded structure with at most four elements in any row. This property allows the use of a special modified Gaussian elimination algorithm for solving the system

(Fread, 1971). Modification of the elimination algorithm reduces the core storage from $4N^2$ to 8N and the number of computational operations are reduced from the order of $(16/3N^3+8N^2)$ to 38N. The increase in computational efficiency is critical to the feasibility of the implicit solution technique.

Boundary Conditions. Known conditions of discharge (Q) or water surface elevation (h) at the upstream and downstream extremities of each river reach for all times ($t=0$ to $t=t_e$, where t_e is the future time at which the simulation ceases) are needed in addition to the initial conditions. In DWOPER the upstream boundary may be a specified discharge or water surface elevation (WSEL) hydrograph for each river. The downstream boundary of tributaries must always be a WSEL hydrograph which is generated by the model. On the main stem, the downstream boundary condition may be a WSEL hydrograph, discharge hydrograph, or a known relationship between the WSEL and discharge such as a rating curve. In the case of a rating curve boundary condition, the rating may be single-valued and specified as tabular (piece-wise linear) values of WSEL and discharge with linear interpolation provided for intermediate values. The rating may also be a loop rating curve generated internally from cross section and roughness properties of the downstream boundary and the instantaneous water surface slope at the previous time step.

Additional Features. In addition to the various boundary conditions, DWOPER has a number of features which make it applicable to a variety of natural river systems for real-time forecasting. It is designed to accommodate irregular cross-sections located at unequal distances along a single multiple-reach river or several such rivers forming a dendritic or 1st order tree-type configuration. It allows for roughness parameters to vary with location and WSEL or discharge. Temporally varying inflows, wind effects, bridge effects, off-channel storage, levee overtopping and/or crevasse flow are included among its features. An efficient automatic calibration procedure for determining optimum Manning n - WSEL or discharge relationships from observed data is also provided as an option in DWOPER (Fread and Smith, 1978).

Data Management Module. Data handling requirements for day-to-day river forecasting are minimal due to extensive data management features (Smith, 1978) utilizing disk storage. However, preparation of the data for simulation of a river system requires a substantial amount of work. The river system configuration, cross-sections, etc. must be determined and input. Also, stage and discharge data for the boundary and initial conditions must be determined and input. This initial work cannot be avoided; however, the data management module does substantially reduce the time and effort required to use the model on a day-to-day operational basis. The data initially input to simulate a particular river system are kept on disk storage and only the updated information for boundary conditions need be input before a new simulation can be made.

The data stored on disk are of two types: stationary data which does not change with time and stage-discharge data which must be updated as new observations are reported. The stationary data are stored in "carryover" files and stage-discharge data are stored in "hydrograph" files. In order to perform a forecasting run, the river system configuration and physical properties must be determined by retrieving the data in a carryover file and the stage-discharge data must be retrieved from a hydrograph file. After the initial simulation run, the initial conditions which consist of the stages and discharges at every computational point in the river system are available in the carryover file as computed stages and discharges which have been stored from a previous run.

The data management module element and the dynamic wave computational element are accessed by commands. Each command causes the program to branch to an appropriate subroutine where the desired function is performed. Some of the more significant data management commands are:

1) COINIT - The carryover file is initialized, i.e., the river configuration, cross-section properties, etc., are input.
2) HINIT - The hydrograph file is initialized, i.e., the hydrograph values are input.
3) COEDIT - Any stationary data contained in the carryover file may be updated by simple reference indicators input along with the updated value(s).
4. HEDIT - Any data contained in the hydrograph file may be deleted, replaced, or added to by simple label and time period indicators input along with the new hydrograph values.
5) COLIST - List the contents of a particular carryover file.
6) HLIST - List the contents of a particular hydrograph file.

Some of the commands used to activate the dynamic wave computational element are:

1) RUN - Simulates a river system using data from carryover and hydrograph files stored on disk.
2) ICSAVE - A command used concurrently with RUN command. ICSAVE is used to save the water surface elevations and discharges at all computational points at a specified time. These values are retained in disk storage for use in subsequent simulation runs as the appropriate and necessary initial conditions.
3) ALONE - Simulates a river system using data (river system configuration, cross-sectional properties, hydrograph values at boundaries, etc.) input at the same time as the ALONE command is input.

OCEAN-RIVER INTERFACE

Ocean Dissipative Effects. In coastal rivers, the channels tend to have very flat slopes of less than two ft/mile and wide floodplains which are usually protected by levees. When a flood wave is routed down the channel and the tidal effects are minimal as in rivers flowing into the Gulf of Mexico, the effects of the ocean tide may be ignored and the flood wave dissipates rapidly as it propagates into the ocean. Real-time forecasting for this condition occurs for a reach of the lower Mississippi River extending from Vicksburg (river mile 437.0) to a section in the Gulf of Mexico which is 23 miles below the Head of Passes (see Fig. 2). The downstream boundary condition of a specified time history of elevation should be for a section so located that the river flood does not significantly affect it. Thus, the downstream boundary for the Mississippi River is located approximately 10 miles into the Gulf as shown if Fig. 3. The boundary condition is considered to be a constant water elevation (mean sea level) with time. If this condition were imposed for a river section at the Head of Passes, the flood wave at sections in the upstream vicinity would be erroneously underpredicted as shown in Fig. 4 for the Venice section.

To help control flooding on the Mississippi Rivers, diversion control structures are used to divert water from the main channel. The lower Mississippi is equipped with three such structures - Old River Diversion, Morganza Diversion, and Bonnet Carre Diversion. The flow diverted to these structures may be input into the DWOPER model as lateral outflow hydrographs. DWOPER also has the capability of computing the flow diverted from the channel by specification of the percentage of the flow to be diverted as a function of time.

This reach of the lower Mississippi River is contained within levees for most of the length, although some overbank flows occur along portions of the upper 210 miles. The average channel bottom slope is a very mild 0.15 ft/mi. A total of 42 cross sections located at unequal intervals ranging from 2-30 miles were used to describe the 460 mile reach.

The reach was first automatically calibrated by DWOPER for the 1969 spring flood. The time steps of 24 hours were used. The gaging station stations used in the calibration were Baton Rouge, Donaldsonville, Reserve, Carrollton, Chalmette, and Pt. a la Hache. Then, using the calibrated set of Manning n vs. discharge values for each reach bounded by gaging stations, the 1969 flood was simulated using stage hydrographs for upstream and downstream boundaries at Red River Landing, and Venice, respectively. The simulated stage hydrographs at the intermediate gaging stations were compared with the observed values. The root-mean-square (RMS) error was used as a statistical measure of the accuracy of the calibration. The RMS error varied from 0.17-0.36 ft. with an average value of 0.25 ft.

Several historical floods from the period 1959-1971 were then simulated using the calibrated Manning n values obtained from the 1969 flood. The RMS error for all the floods was 0.47 ft. This compared with 0.25 ft. for the calibrated flood of 1969, indicating that this reach of the Mississippi River there is not a significant change in the channel roughness from one flood event to another. The simulated vs. observed WSEL in 1969 and 1966, respectively, for the Baton Rouge and Carrollton gaging stations are shown in Figs. 5-6. The average RMS error for all gaging stations in the simulation of the 1966 flood was 0.38 ft.

In 1977 the reach was extended up to Vicksburg and down to a section 23 miles below Head of Passes. This reach was later calibrated using the 1977 flood, Vicksburg and the Gulf as its upstream and downstream boundary conditions, and six intermediate gaging stations - Natchez Landing, Red River Landing, Baton Rouge, Donaldsonville, Reserve, and Carrollton. The results were similar to those calibrated by the 1969 flood.

DWOPER requires approximately 0.001 CPU seconds per Δx per Δt on the NAS 9050 computer. Since time steps of 24 hours are used, the total CPU time required to forecast a typical flood through the 460 mile reach of the Mississippi River from Vicksburg to the Gulf is 2.8 seconds.

Ocean Tidal Effects. DWOPER is currently being used for real-time forecasting on the 130 mile reach of the lower Columbia River below Bonneville Dam, including the 25-mile tributary reach of the lower Willamette River. A schematic of the river system is shown in Fig. 7. The downstream boundary for this system is a tide hydrograph. These tides are obtained from a predicted tide table produced by the National Ocean Service. During a forecast period the tide values are updated manually to reflect current observations.

This reach of the Columbia has a very flat slope (0.06 ft/mi) and the flows are affected by tides from the Pacific Ocean. The tidal effect extends as far upstream as the tailwater of Bonneville Dam during periods of low flow. Reversals in discharge during low flow are possible as far upstream as Vancouver. A total of 25 cross sections located at unequal distance intervals ranging from 0.5-12 miles are used to describe the river system. One hour time steps are used in simulations.

The system was first calibrated for a 4-day period in August 1973. Seven intermediate gaging stations at Warrendale, Washougal, Vancouver, Portland, Columbia, Rainier, and Wauna were used along with the gaging stations at the extremities of the system, i.e., Bonneville, Oregon Falls, and Tongue Pt. Another 5-day period in August 1973 was then simulated using DWOPER and the calibrated Manning n - discharge relations. Upstream and downstream boundaries were observed discharges and stages, respectively. The average RMS error for all stations in the simulation hydrographs for Warrendale, Vancouver, Portland, and Wauna range from 0.19 ft. at Warrendale to 0.32 ft. at Wauna. The observed and computed values of WSEL are shown in Fig. 8. Time steps of 1 hour are used when forecasting the lower Columbia River; approximately 1.2 CPU seconds (NAS 9050) are required to forecast for a period of 24 hours.

The DWOPER model has been applied to this river reach for several special projects. It is used to forecast the tidal effects on navigational depths. Sediment blockage of the Columbia River by deposits of mudflows from Cowlitz - Toutle rivers in aftermath of the Mt. St. Helens volcanic eruption was also forecasted. The model is also used on this river to compute travel times for chemical and oil spills.

Storm Surges. Real-time forecasting of river flooding due to hurricane-produced storm surges is presently in operation for the lower portions of the Mississippi, Sabine, Neches, Trinity, San Jacinto, Brazos, Colorado, Guadalupe, Lavaca, Navidad, Nueces and Rio Grande rivers. To forecast this type of flooding, two hydrodynamic models are used. Figures 9 and 10 illustrate the use of the two models (SLOSH and DWOPER) for the lower Mississippi, Sabine and Neches rivers are modelled.

The hurricane-generated storm surge is predicted using the SLOSH model (Jelesnianski, 1967, 1972, 1976) which is a two-dimensional, vertically integrated hydrodynamic model. Externally specified meteorological parameters are utilized to generate the hurricane wind field. These parameters are the radial distance and pressure drop from the storm center to its periphery, and the forward speed of the storm. The wind field submodel empirically computes the maximum wind speed in a stationary storm and generates the wind field by dynamically balancing the computed wind speed, pressure gradient, and inflow angle fields. The computed wind field is then incorporated into the two-dimensional hydrodynamic equations through the wind stress term which drives the model, i.e., causes the development of the storm surge. The SLOSH model has the capability to treat overtopping of finite barrier heights to allow coastal flooding. SLOSH provides predictions of water surface elevation on a polar coordinate two-dimensional grid.

The one-dimensional hydrodynamic model, DWOPER, uses the time history of the WSEL predicted by the SLOSH model due to the storm surge at the mouth of the river as its downstream boundary condition and a specified discharge hydrograph as its upstream boundary. The upstream boundary is located considerably beyond the last point of interest where it is assumed the surge has insignificant effect on the specified discharge. The coupling between SLOSH and DWOPER is external in the sense that any downstream flood in propagation along the river is not treated during the SLOSH computations. Although it is recognized that such external coupling of the two models is not ideal, it is nevertheless considered the practical choice due to such factors as (a) dampening of the coupling effect as the surge propagates further upstream; (b) the models were developed, maintained, and operationally used by three separate divisions of NWS; and (c) the real-time use of the models.

An example of the DWOPER model's ability to simulate storm surges moving upriver was determined for the 1969 Hurricane Camille which produced a strong surge with water levels up to 12 ft. in the Mississippi Delta area. This hurricane surge propagated into the lower Mississippi River and traveled upriver several hundred miles. The DWOPER model was used to compute the WSEL's and discharges produced by the passage of the surge at Chalmette (river mile 91.0) and Carrollton (river mile 102.8). The downstream boundary condition was the observed hourly stage hydrograph at Pointe a la Hache (river mile 46.7) and the upstream boundary condition was an assumed steady discharge of 253,000 cfs at Red River Landing (river mile 302.4). The Manning roughness coefficients for the study reach were maintained the same as determined during the calibration of the 1963 flood. The computed and observed WSEL hydrographs at Chalmette and Carrollton are shown in Fig. 11. The RMS errors between the computed and observed stage hydrographs are 0.33 and 0.34 ft. for Chalmette and Carrollton, respectively. Time steps of 1 hour are used when forecasting hurricane surges in the lower Mississippi River. A typical simulation requires approximately 1.5 CPU seconds (NAS 9050).

Real-time forecasting using SLOSH and DWOPER was used to predict the impact of Hurricane Bob in 1978 with prediction errors of less than one foot in the vicinity of New Orleans.

SUMMARY AND CONCLUSIONS

A one-dimensional, implicit hydrodynamic model (DWOPER) is used by the National Weather Service for real-time flood forecasting on several coastal rivers in the United States. The model is based on the complete unsteady flow equations. A weighted four-point nonlinear implicit finite difference scheme is used to obtain solutions to the unsteady flow equations via a Newton-Raphson iterative technique. DWOPER has several features which make it applicable to a variety of natural systems for real-time forecasting. It is designed to accommodate various boundary conditions and irregular cross sections located at unequal distance intervals along a single multiple-reach river or several such rivers having a dendritic configuration. Data handling requirements for day-to-day river forecasting are minimal due to extensive data management features utilizing disk storage. Operationally, data input is only required to update hydrograph files with the most recent observations. Applications of DWOPER to several coastal river systems have demonstrated its operational efficiency, accuracy, and utility.

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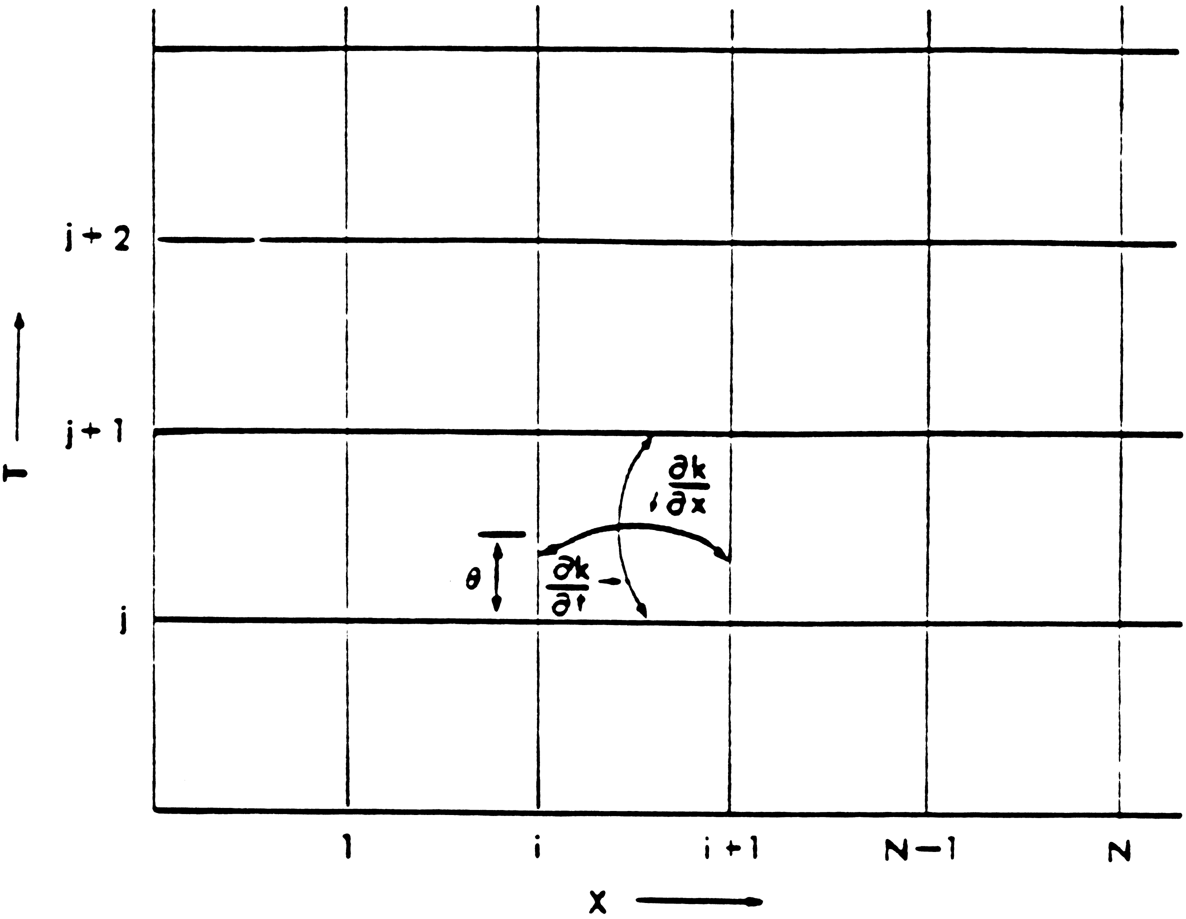

FIG. 1 - DISCRETE X-T SOLUTION DOMAIN

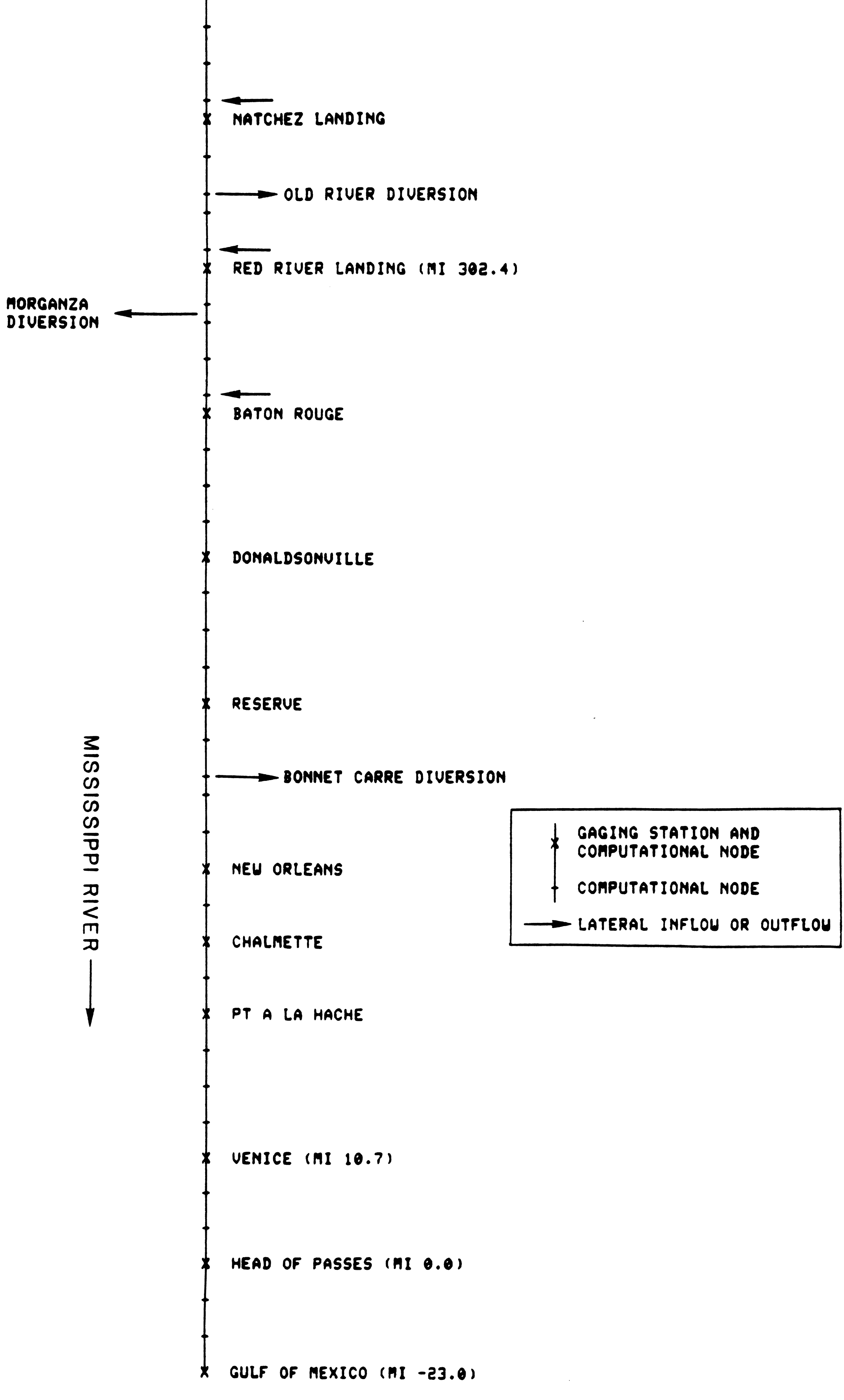

FIG. 2 SCHEMATIC OF LOWER MISSISSIPPI RIVER

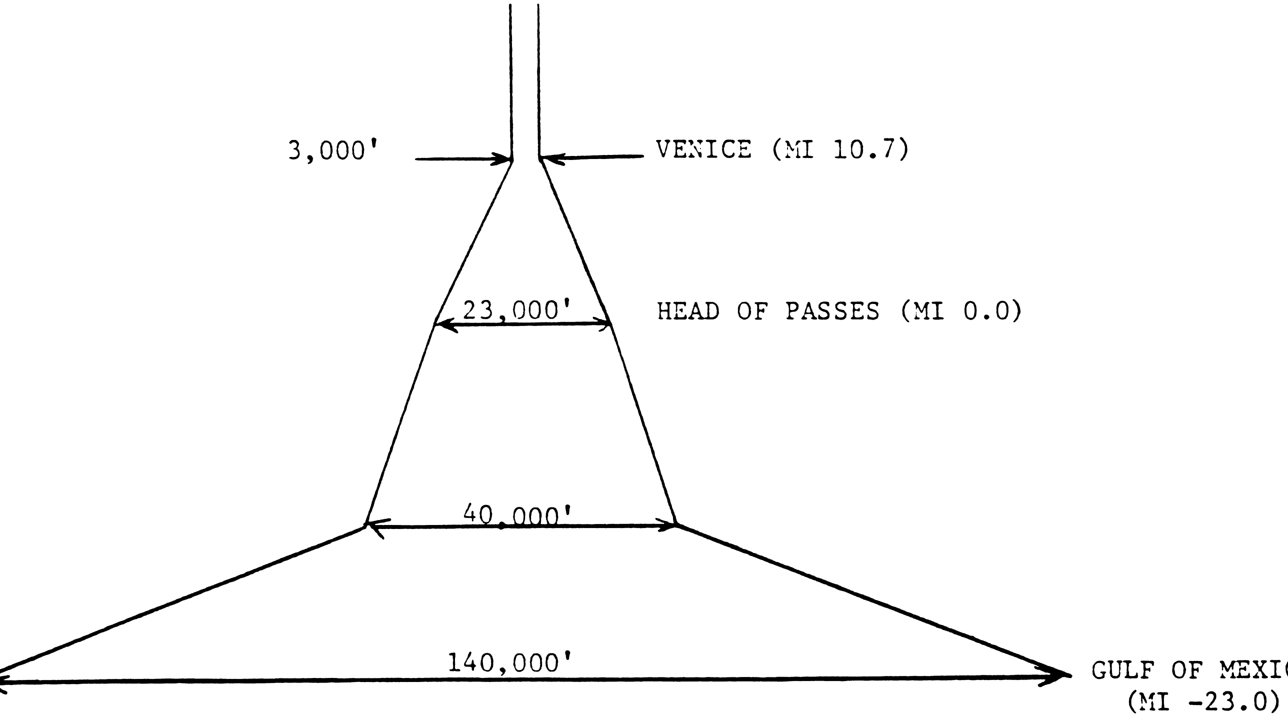

FIG. 3 - PLAN VIEW OF LOWER MISSISSIPPI RIVER - OCEAN INTERFACE

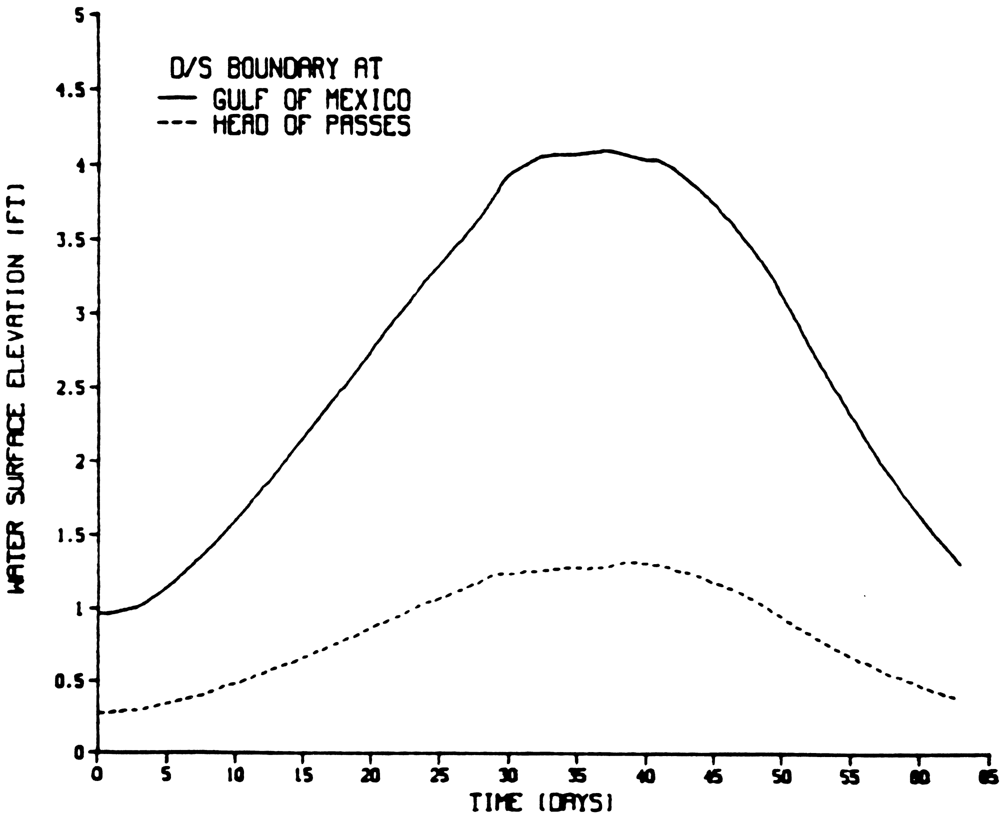

FIG. 4 - WATER SURFACE ELEVATION HYDROGRAPH AT VENICE

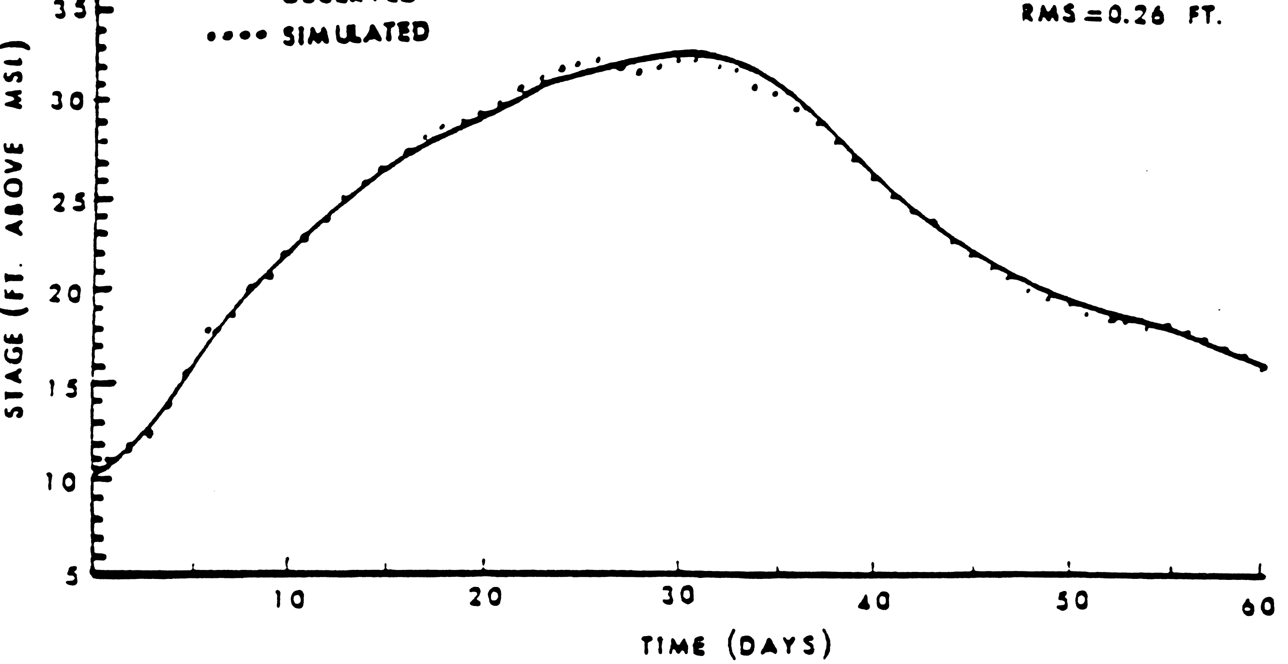

FIG. 5a - OBSERVED VS. SIMULATED STAGES AT BATON ROUGE FOR 1969 FLOOD

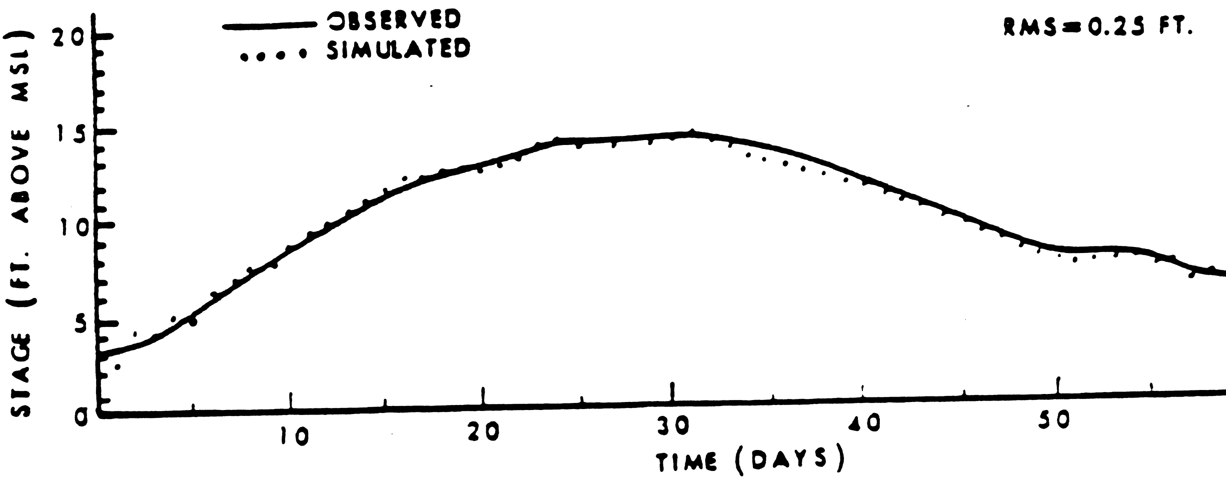

FIG. 5b - OBSERVED VS. SIMULATED STAGES AT CARROLLTON (NEW ORLEANS) FOR 1969 FLOOD

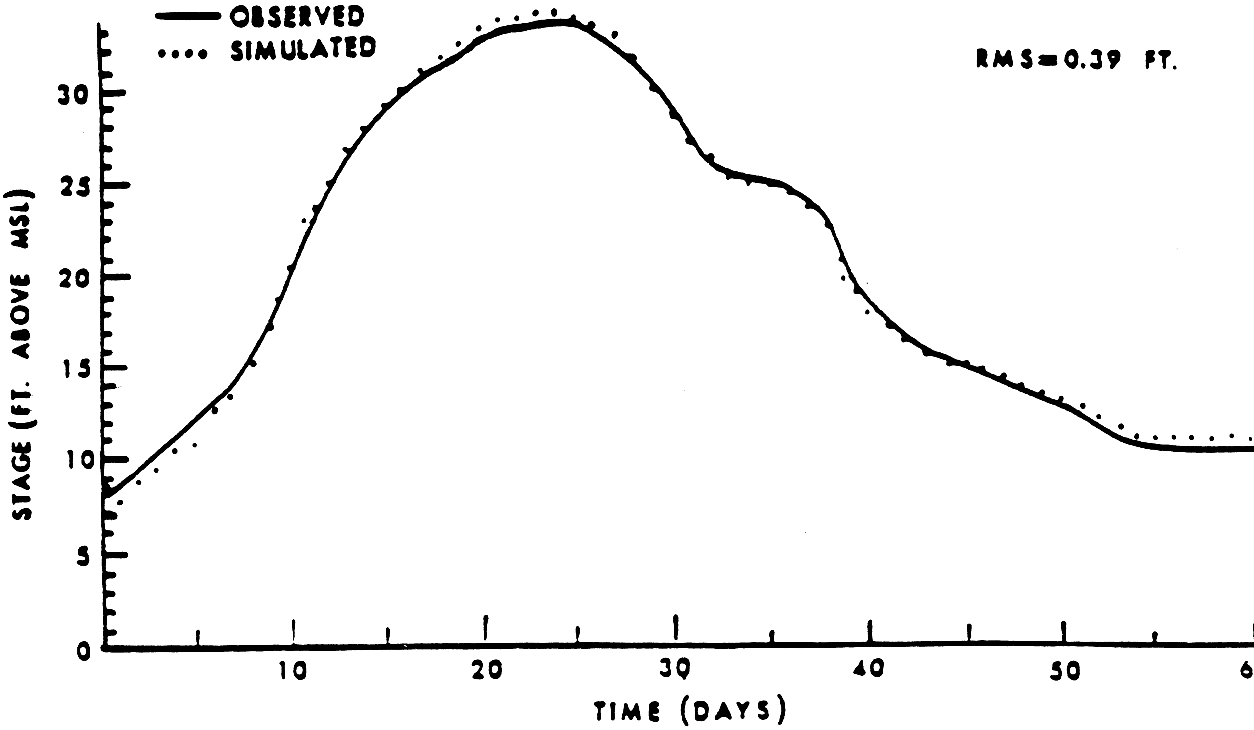

FIG. 6a - OBSERVED VS. SIMULATED STAGES AT BATON ROUGE FOR 1966 FLOOD

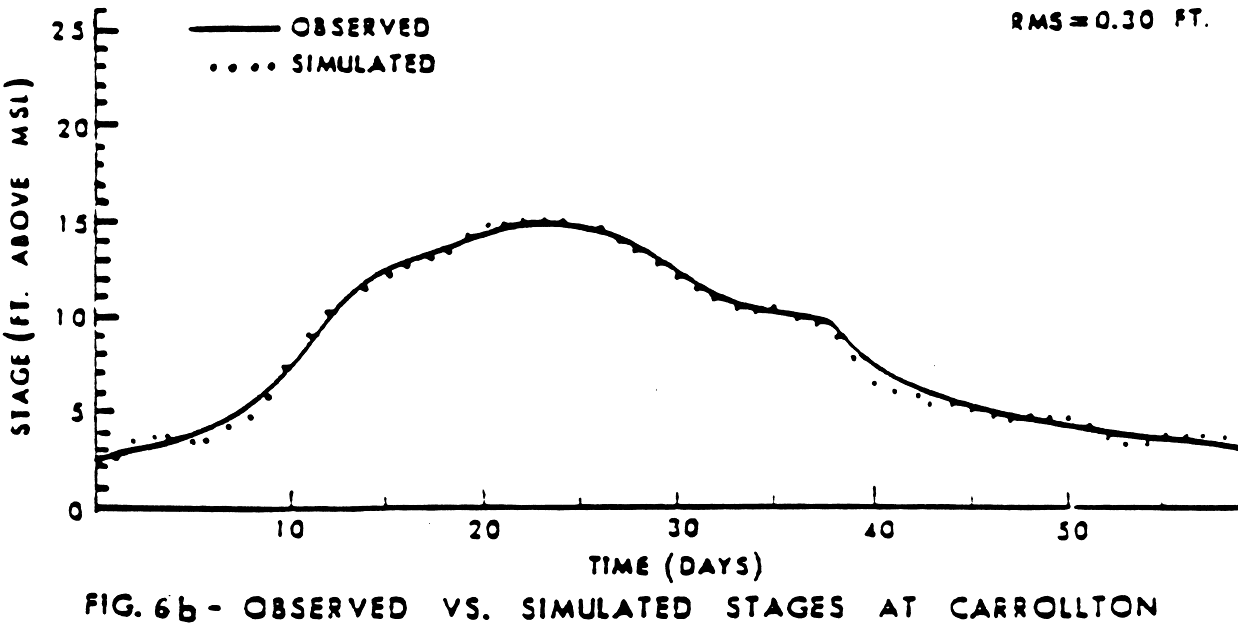

FIG. 6b - OBSERVED VS. SIMULATED STAGES AT CARROLLTON (NEW ORLEANS) FOR 1966 FLOOD

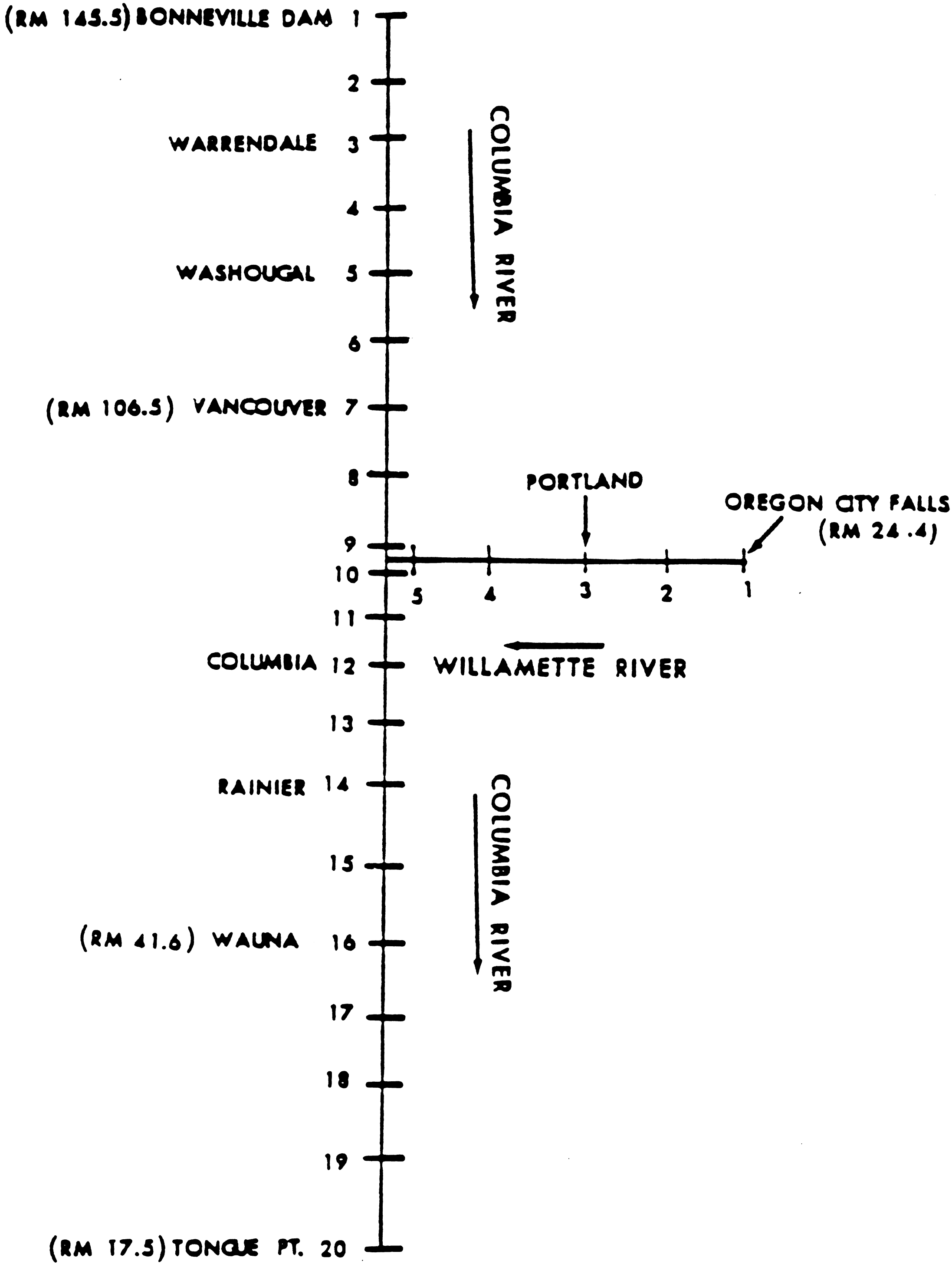

FIG. 7 - SCHEMATIC LOWER COLUMBIA-WILLAMETTE RIVER SYSTEM

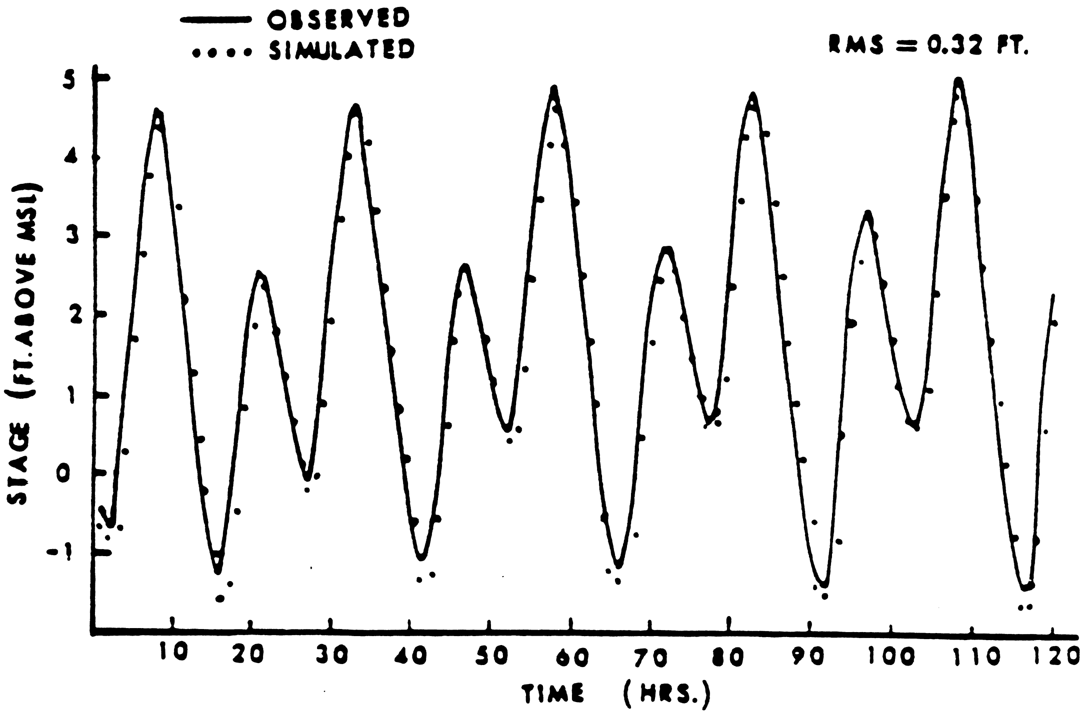

FIG. 8a - OBSERVED VS. SIMULATED STAGES AT WAUNA.

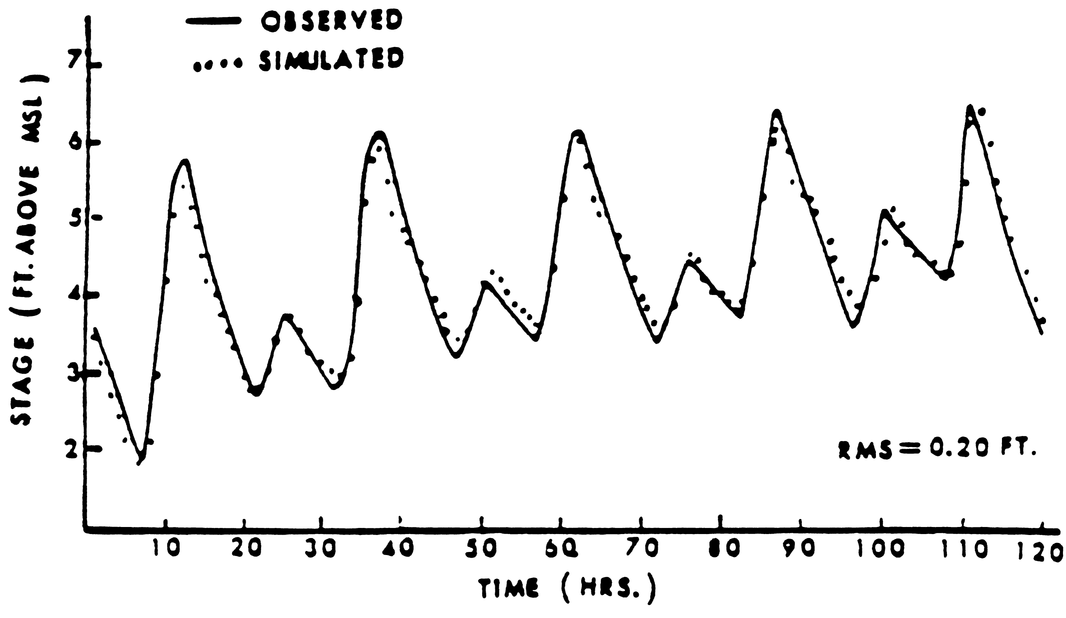

FIG. 8b - OBSERVED VS. SIMULATED STAGES AT PORTLAND

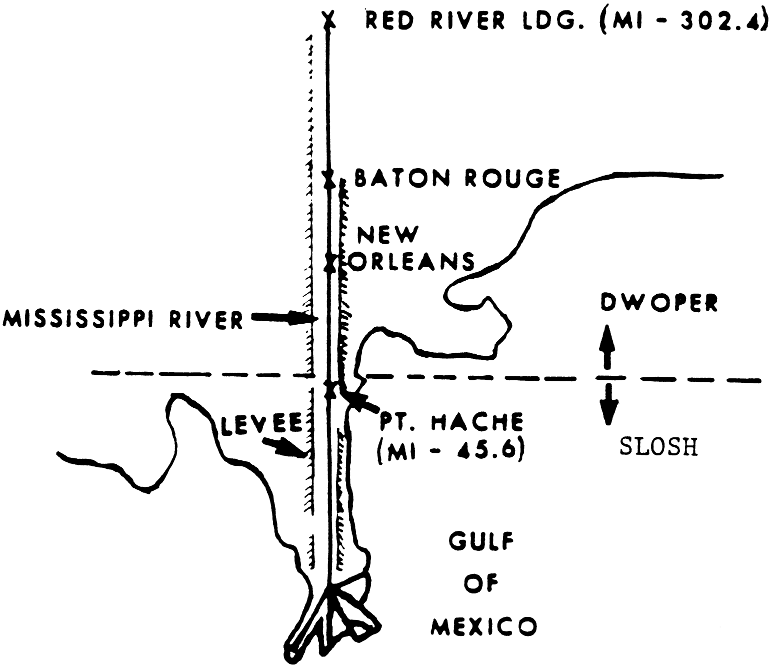

FIG. 9 - HURRICANE STORM SURGE FORECASTING OF LOWER MISSISSIPPI RIVE

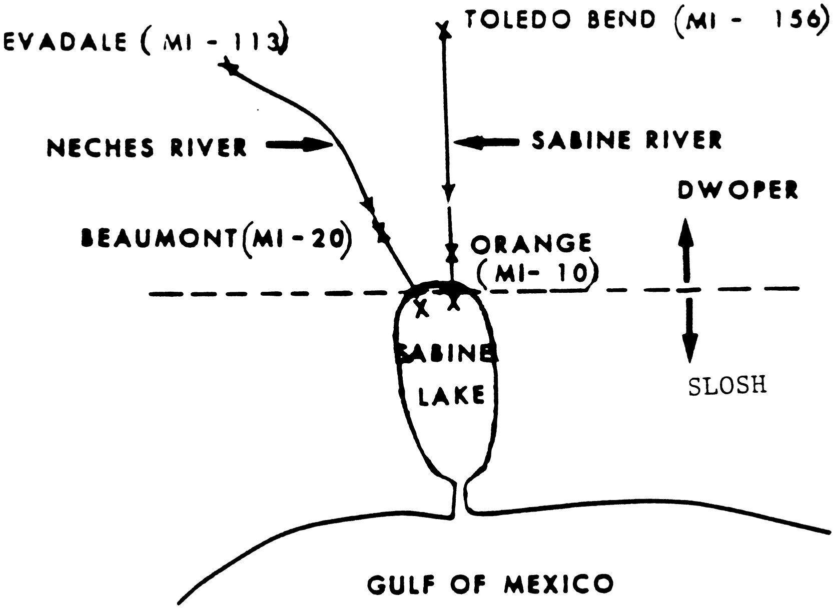

FIG. 10 - HURRICANE STORM SURGE FORECASTING OF SABINE AND NECHES RIVERS

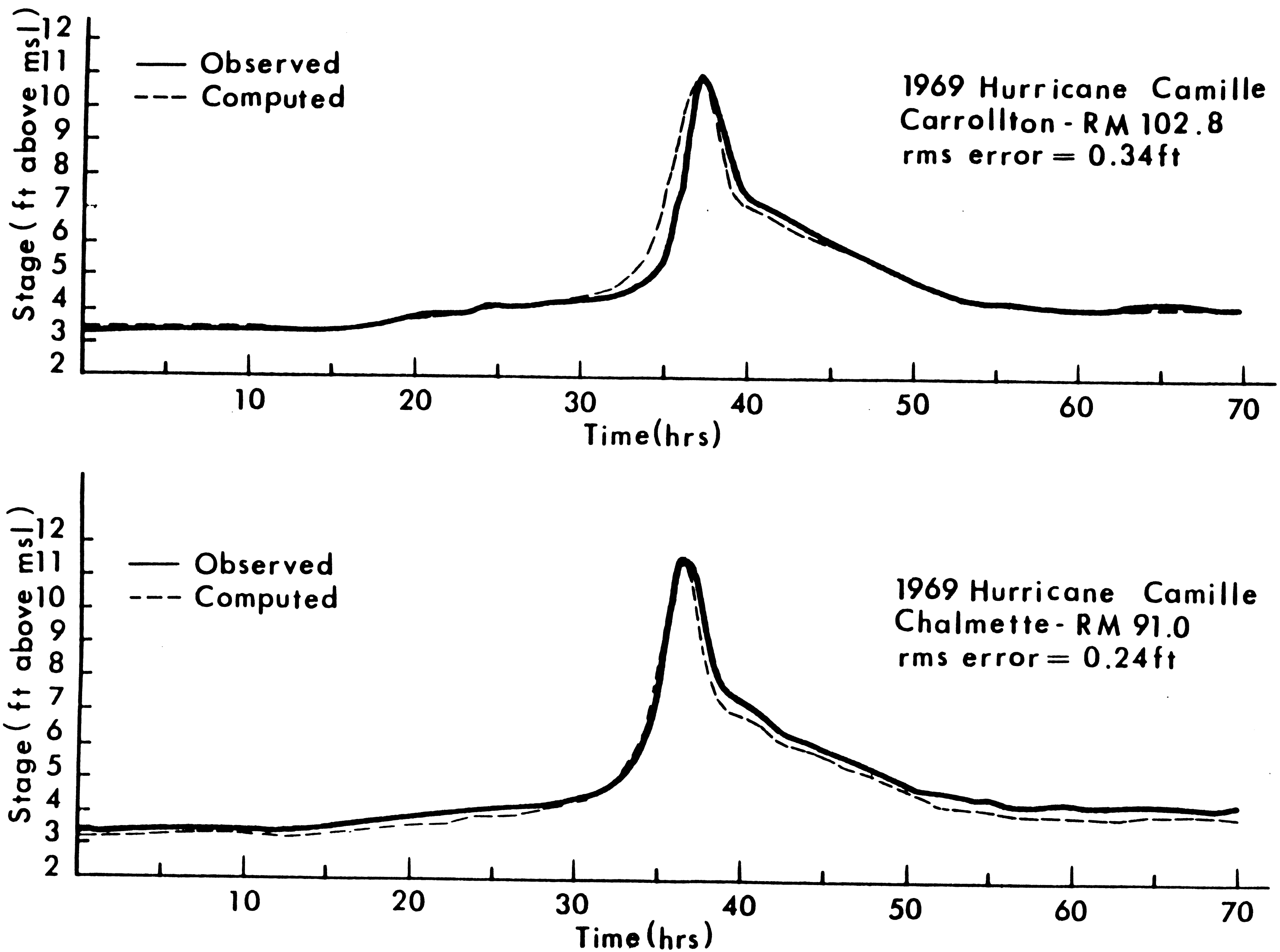

FIG. 11 - STAGE HYDROGRAPHS FOR 1969 HURRICANE CAMILLE AT CHALMETTE AND CARROLLTON